Restoration of the Salton Sea

Volume 2: Embankment Designs and Optimization Study

Appendix 2C: Deformation Analyses

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1.0 Introduction

This report presents the interim results of the geotechnical analyses of seismically induced deformations using the explicit finite difference code Fast Lagrangian Analysis of Continua (FLAC) and a combination of Newmark and Makdisi-Seed displacement methods. This report forms Appendix 2C in Kleinfelder's complete report for the Salton Sea restoration project.

1.1 Purpose

The purpose of the FLAC analyses presented in this report were to verify the deformation performance of the alternative structures, assist in the Risk Analysis, and to also assist in the design optimization process for both the preferred mid-Sea, north-Sea, and south-Sea dams and perimeter dikes configurations. As such, as described in other locations in the main report, the sand dam with stone columns embankment option was identified as the preferred configuration. The FLAC models described in this report are of the mid-Sea "Sand Dam" option and Reclamation's initial perimeter dikes option.

1.2 Scope of Work (Task 8)

Perform 2-dimensional FLAC deformation analyses for each of the four (4) optimized mid-Sea dam and perimeter dike cross-sections (the dam embankment and perimeter dikes, with and without foundation liquefaction assumed). Material properties were developed for each section in consultation with Reclamation. Kleinfelder met with Reclamation and jointly determined the appropriate earthquake time histories and return periods to be used in these analyses. These analyses were performed by using the regular polarity of each time history provided by Reclamation, with results reported for both direct and reverse polarity. A total of four (4) analyses were performed. These analyses included the optimized mid-Sea dam configuration and Reclamation's initial perimeter dike cross-section using a reservoir elevation 5 feet below the embankment crest on the upstream side of the dam and dike.

1.3 Authorization

The scope of work outlined above was performed based on authorization by Reclamation under Order No. 04B8810942 of Contract No. 04CA810942, dated April 21, 2006, between the Bureau of Reclamation and Samuel Engineering, Inc. of Englewood, Colorado. Kleinfelder has performed the work summarized in this report under subcontract agreement with Samuel Engineering.

1.4 Project Personnel

The following personnel from Kleinfelder performed the work described in this report:

Project Manager:	Keith A. Ferguson, P.E.
Principal Investigator/Team Leader:	Zia Zafir, Ph.D., C.E., G.E.
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Staff Engineers:	Scott T. Anderson, P.E Jorge Meneses, Ph.D, P.E. Jie Yu, P.E.

1.5 Acknowledgements

Reclamation personnel who assisted with and supported the efforts in completing this study included: Robert Dewey, Karl Dise, Perry Hensley, David Gillette, Paul Weghorst, and Richard Wiltshire. The input from and support of Reclamation are gratefully acknowledged.

2.0 Design Earthquake and Design Input Motions

Design ground motions for this project consisting of uniform hazard spectra (UHS) and spectra-matched time histories were developed by Reclamation. In addition, Reclamation performed site response deconvolution analyses to develop two sets (each set containing two horizontal and one vertical motions) of time histories at the base of the FLAC model (El –418) to be used as the input in our FLAC analyses.

2.1 Uniform Hazard Spectra (UHS)

Site-specific horizontal and vertical UHS at the ground surface were developed for return periods of 10,000, 5,000, 2,500, and 500 years. These spectra were developed using probabilistic methods. Seismic source model and other details about the analyses will be presented in a separate report by Reclamation. Per discussions with Reclamation, the ground motions representing an event having a return period of 10,000 years are the design ground motions for this project. The 10,000-year horizontal and vertical ground surface UHS are presented in both linear and tripartite format in Figure C.2.1. These spectra are the target spectra to spectra match the time histories. Results of deaggregation analyses by Reclamation indicate that the dominant magnitude and distance are 7.4 and less than 10 km, respectively for the 10,000-year event.

2.2 Time Histories

Reclamation selected two sets of historical recorded time histories and then matched them with the target spectra. Both of these sets were obtained from the 1992 (M7.3) Landers earthquake. Each set of time history contains two orthogonal horizontal motions and one vertical motion. Horizontal and vertical motions were matched with horizontal and vertical target spectra, respectively. Pertinent characteristics of recorded time histories used in the spectra-matching routine are presented in Table 2C.2.1.

Spectra-matched acceleration (g), velocity (ft/sec), and displacement (ft) time histories and their response spectra in linear and tripartite forms are presented in Figures C.2.2 through C.2.9. These time histories were baseline corrected in the time domain by Reclamation.

Table 2C.2.1: Earthquake Time Histories for Deconvolution Analysis

Motion	Site Class	Dist. (km)	Orientation	PGA (g)	PGV (ft/sec)	PGD (ft)
1992 Landers - Palm Springs Airport	D	37.5	90º 360º Up	0.089 0.076 0.108	0.453 0.358 0.223	0.174 0.228 0.101
1992 Landers – Desert Hot Springs	C	23.2	90º 360º Up	0.154 0.171 0.167	0.686 0.663 0.325	0.255 0.455 0.122

Note: Dist. refers to closest distance to fault rupture.

2.3 Deconvolution

These spectra-matched time histories were then used as the input in the deconvolution analyses to develop input motions at the base of the FLAC model (El –418). Deconvolution analyses were performed using an in-house computer program by Reclamation. The deconvolved acceleration (g), velocity (ft/sec), and displacement (ft) time histories and their response spectra are plotted in Figures D.2.10 through D.2.17. These time histories were baseline corrected in the time domain by Reclamation.

3.0 Sliding Block Analyses (Newmark)

During the initial stages of seepage and stability evaluations of the various cross-section options under consideration, a concern arose with respect to seismic deformations. Reasonably conservative sections having "post-earthquake" factors of safety significantly higher than 1.3 (Reclamation's specified design criteria) had low to very low yield accelerations. Deformation vs. yield acceleration curves developed and presented by California Department of Water Resources (2005) suggested that deformations well in excess of the 5 feet of freeboard would occur. The first attempt to resolve this issue was to perform stability evaluations of deformed dam sections to identify the changes to yield acceleration that would occur. These analyses showed some increase in the yield acceleration. However, for deformations in the range of available freeboard, the changes in the estimated yield acceleration were not substantive enough to indicate that the deformations would be less than or equal to the design criteria (5 feet).

Subsequently, simplified Newmark and Makdisi-Seed deformation analyses were performed for a variety of constant yield accelerations utilizing surface and deconvolved ground motions for the site (see Chapter 2.0). The USGS program NEWMARK (USGS, 2003) was used to perform the Newmark portion of these analyses. These results conservatively indicated that all cross-sections would need to have a yield acceleration of between 0.15 and 0.20g in order to limit crest deformations to less than the embankment freeboard. Subsequently, cross-sections were modified to produce yield accelerations in this range. The "post-earthquake" factor of safety for these sections typically ranged from 2.4 to 2.8 (See Appendix 2B).

The results of the Newmark and Makdisi-Seed deformation analyses are plotted on Figures C.3.1, through C.3.4. Figures C.3.1 and C.3.2 show the Newmark displacements calculated from the 10,000-year input motion vs. yield acceleration while Figures C.3.3 and C.3.4 show the predicted Makdisi-Seed displacements. Both the surface and deconvolved motions are plotted in these figures. For the Newmark analyses, the maximum and mean motions are plotted and for the Makdisi-Seed analyses, only the mean motions are plotted. The moment magnitude for the Makdisi-Seed analysis was taken as 7.4, as described in Chapter 2.0.

Also plotted on these figures are the estimated minimum and average vertical crest displacements that were determined with FLAC. Further discussion of the FLAC modeling results and comparison of the FLAC results with the deformations estimated using the Newmark and Makdisi-Seed methods is presented in subsequent chapters.

The Newmark and Makdisi-Seed approaches are not to be used to verify the adequacy of a design if the available freeboard is less than 3 feet (perimeter dike) and/or if embankment or foundation materials may liquefy (Reclamation, 2001). However, it is noted here that the simplified deformation analysis methods combined with the FLAC results provided a compelling basis to establish a yield acceleration criterion (0.17g) that would successfully limit deformations to within design criteria limits.

4.0 FLAC Models and Material Properties

This chapter describes the development of the FLAC models and material properties used in the FLAC analyses. A summary of the study analysis cases results, and guide to Figure numbers is presented in Table 2C.4.1. A summary of the material properties and the general geometry of each of the alternative sections considered in these analyses are provided in Tables 2C.4.2, 2C.4.3, 2C.4.4, and 2C.4.5 and shown on Figures C.4.1 and C.4.2. Note that the water surface and crest elevations shown below may not be consistent with final elevations presented in the main report.

4.1 Configuration of the Preferred Mid-Sea Dam and Perimeter Dike Options

The overall configuration of the mid-Sea dam and perimeter dike cross-sections selected by Reclamation for FLAC analyses are described below. All elevations are in feet and referenced to mean sea level (MSL).

4.1.1 Mid-Sea Dam

The sand dam with stone columns cross-section option (Figure C.4.1) would be constructed by first removing all Seafloor deposits from beneath the dam footprint (to elevation –280) and the soft lacustrine and/or upper alluvial deposits from a 372-foot-wide area beneath the central core area. For purposes of developing the FLAC model, it was assumed that the base of the soft lacustrine materials was located at an elevation of approximately -305 feet. The central core of sand/gravel embankment fill (Type A) would be improved with stone columns so that these materials would have an equivalent $N_{1,60}$ blowcount of greater than 20. For the purposes of analysis, the Type A material that forms the core was sloped at 3 (horizontal) to 1 (vertical) (3H:1V) on the upstream and downstream sides, with a crest width of 30 feet at an elevation of -223 feet.

The outer shells of the dam would consist of a sand/gravel (Type B) material placed at a 10H:1V slope (both upstream and downstream) from the crest continuing downward to an elevation of about -268 feet. A soil-cement-bentonite (SCB) wall would be located along the centerline of the dam and have an approximate width of 5 feet and keyed into the underlying upper stiff lacustrine soils. For the purposes of these analyses, the SCB wall was not explicitly modeled. The soft lacustrine soils are underlain by upper stiff lacustrine soils to an unknown depth. The slopes of the outer shells of Type B material would be armored with riprap material; this material was not explicitly modeled in the analysis. Instead, the outer portion of the embankment was given a small amount of tensile strength to reflect the stronger nature of the riprap and to provide numerical stability.

4.1.2 Perimeter Dikes

The construction of the perimeter dikes also consists of the removal of the Seafloor deposits within the footprint of the dike (to elevation -264). The core material would consist of Type A sand/gravel having a width of approximately 15 feet at the base (elevation –264) and narrowing to a width of 5 feet at the crest elevation of –239. Fine rockfill would be placed between the core and the outer rockfill shells. The outer slope of the fine rockfill would be 1.5H:1V. The outer shell of rockfill material would be sloped at 4H:1V. Reclamation's initial perimeter dike concept also included a vinyl sheet pile cutoff wall that would be keyed into the underlying upper stiff lacustrine soils.

The perimeter dikes are underlain by either soft lacustrine or alluvial soil deposits along the planned alignment. Models 3 and 4 both have assumed that alluvial soils underlie the dike and are encountered between elevations -264 and -276 feet. Under the outer shells, the alluvial soils would be jet grouted and extend from the inner/outer shell boundary a distance of approximately 30 feet to the upstream and downstream directions.

4.2 Soil Properties

Soil properties were derived based on the preliminary in-sea exploration program performed by URS (2004a), which included soil borings, cone penetration test (CPT) soundings, and laboratory testing results. The California Department of Water Resources (DWR) report (2005) and URS reports (2004a, 2004b, and 2005) provided additional information for the development of the soils properties. The soil properties were also chosen to be consistent with the previously developed soil properties of the Task 6, Seepage and Stability Analyses summarized in Appendix 2B of Kleinfelder's complete report.

4.2.1 Mid-Sea Dam

Embankment materials were defined as sand/gravel fill (this report includes cross-sections with very specific designations such as Type A sand/gravel, Type B sand/gravel, riprap etc.) for the shell materials and for the core. The Type A sand/gravel materials in the central core of the dam and dikes included gravel in stone columns. This central core of the sand dam with stone columns was modeled with composite properties meant to be representative of the improved stone column sand/gravel matrix. Figure C.4.1 presents the material distribution within the generalized model of the mid-Sea dam embankment cross-section.

The embankment materials were underlain by soft lacustrine soils from elevation -280 to -305 feet and by upper stiff lacustrine soils from -305 to -418 feet. The upper stiff lacustrine clay was divided into three layers with shear wave velocities that increased with depth and K_0 values that decreased with depth. Information on the foundation materials is summarized in Tables 2C.4.2 and 2C.4.3

The Seafloor deposits were not explicitly modeled as the very soft consistency of these materials would lead to problems with numerical stability of the model. The weight provided by this layer of material was replaced by an applied pressure on the top of the soft lacustrine material, as was the weight of the overlying saline water. Due to the salt content of the water, the unit weight of water was taken to be 64.0 lb/ft^3.

The soil-cement-bentonite (SBC) slurry wall was not explicitly modeled. The contribution or lack of contribution of the stiffness of the SBC slurry wall was deemed to be minimal. The phreatic surface was explicitly specified to match the assumptions used in the slope stability and seepage analyses.

A parametric study was conducted on the material properties assumed for the Type A sand/gravel core material improved with stone columns and the Type B sand/gravel shell material. This portion of the study is discussed further in Chapter 6.0.

4.2.2 Perimeter Dikes

Embankment materials for the perimeter dikes were defined as the sand/gravel core (Type A), the fine rockfill, and the rockfill shell as previously described. Figure C.4.2 shows the dimensions of the perimeter dike embankment cross-section.

The perimeter dike model embankment is underlain by alluvial deposits from elevation -270 to -280. The alluvial deposits are underlain by upper stiff lacustrine clay to the depth modeled, elevation -418. Information on the foundation materials is summarized in Tables 2C.4.4 and 2C.4.5.

The vinyl sheet pile seepage cutoff along the centerline axis of the dike was judged to have little influence on the deformation of the perimeter dike and was therefore not included in the model. The phreatic surface was explicitly specified to match the location estimated from seepage analyses described in Appendix B of Kleinfelder’s summary report.

4.3 FLAC Model Makeup

The FLAC finite difference grids developed for the mid-Sea dam and perimeter dike models are shown on Figures C.4.1 and C.4.2, respectively. These grids were developed based on the layering of soils from the slope stability and seepage models. Due to the large number of solution steps involved in the analysis, the double precision version of FLAC was used, as recommended by Itasca (2006) and Peter Cundell (personal communication).

4.4 Boundary Conditions

As previously noted, the base boundary was set at elevation -418.0 for both the mid-Sea dam and perimeter dike models. To compute in-situ stresses in the initial static analysis, the base boundary was fixed both horizontally and vertically and the side boundaries were only fixed horizontally. Accelerations for the previously described input motions were applied in the dynamic portion of the analysis along the base of the model. The horizontal restraints of the side boundaries were released and replaced with the free-field boundaries such that the plane waves propagating upward suffer little distortion. The free-field grid supplies conditions that are similar to those in an infinite model, which allow the use of a smaller cross-section of the dam or dike and reduce computational demand by reducing the number of zones required to construct the model.

4.5 Solution Steps

The following is a summary of the steps required in the analysis to bring the model to static equilibrium and then perform the dynamic analysis.

- The model, without the embankment materials, was brought to static equilibrium with the elevation of the water at -227 feet for the mid-Sea-dam models and -240.5 for the perimeter dike models
- The static stresses were then altered using the assumed values of K_0 that are listed in Tables 2C.4.2 through 2C.4.5.
- The model was cycled to allow these stresses to take effect.
- The construction of the embankment and all associated excavations (dredging) were assumed to take place instantaneously, as the construction sequences of the dam and dikes are not known with any certainty at this time.
- The model was then brought to static equilibrium under the gravity loading of the embankment materials and pressures from the assumed water levels of -227 on the upstream portion of the mid-Sea dam and -240.5 in the perimeter dike model. The tailwater elevation on the downstream side of the mid-Sea-dam was modeled at an elevation of -268 and on the perimeter dike was modeled at an elevation of -255.
- At this point, the models were ready for the dynamic portion of the analysis, and the models either continued to run with the static properties assumed at the beginning of the static analysis, as in Models 1 and 3, or were changed to liquefied or softened strengths.

- The input motion(s) were applied to the base of the model and run for the total time history record length, approximately 80 seconds.

Several locations were chosen to take histories of key outputs such as, displacement, velocity, acceleration (in both x and y directions), shear stress, and shear strain increment.

5.0 FLAC Dynamic Analysis Results

5.1 Organization of the Analysis Results

In this chapter, a summary of the basic analysis approach used will be presented, followed by a brief discussion of the results obtained for each of the four models outlined in the original scope of work. A parametric analysis was conducted using Model 2. A discussion of the results of the parametric analysis is presented in Chapter 6.0. Graphical results have been prepared and consist of contour plots of x (horizontal) and y (vertical) displacement; contours of maximum shear strain increment; deformed (with magnification of 5 times) vs. un-deformed grid; and graphs with curves for the downstream crest, mid-slope and toe of slope displacement vs. time for both horizontal (x) and vertical (y) directions. A summary of maximum displacement vector, average crest displacement and minimum crest displacement are also given in Table 2C.4.1. The figure numbering convention for these graphical results is as follows:

- Figure C.5.4.1.x – series: Plots of the results from Model 1
- Figure C.5.4.2.x – series: Plots of the results from Model 2
- Figure C.5.4.3.x – series: Plots of the results from Model 3
- Figure C.5.4.4.x – series: Plots of the results from Model 4

5.2 Analysis Approach

The FLAC deformation analyses were performed as a total stress analysis using the Mohr-Coulomb constitutive model. The reasoning behind the use of the total stress analysis approach consisted of several factors: 1) the lack of high quality testing for the determination of material parameters, in particular the lack of dynamic testing to determine the effect of cyclic stresses on the reduction in shear strength and possible pore pressure increases; 2) discussions with the Bureau of Reclamation concerning the use of effective versus total stress analysis; and 3) the current state of knowledge concerning the material parameters. The Mohr-Coulomb constitutive model is the most widely applied material model used for geomechanical modeling; thus, it has a wide range of correlations with which to assist in selecting the material properties. The Mohr-Coulomb model is a linear elastic-perfectly plastic model so that the equations of elasticity can be used to determine the basic stiffness parameters, the shear and bulk modulus.

Other key assumptions included in the evaluation included:

- The stiffness of the material modeled was estimated based on elastic relations to assumed shear wave velocities expressed by the following relationships;

$$V_s = \sqrt{\frac{G}{\rho}}$$

$$V_p = \sqrt{\frac{K + 4G/3}{\rho}}$$

  Where,

  V_s is the shear wave velocity (ft/s)
  V_p is the compression wave velocity (ft/s)
  G is the shear modulus (lb/ft^2)
  K is the bulk modulus (lb/ft^2)
  ρ is the bulk density (slugs/ft^3)

- The material stiffness, and shear modulus were held constant throughout seismic shaking, and
- Liquefied or softened strength properties were applied from the beginning of the time-history.

Assumptions concerning material properties were, as previously stated, based on the California DWR report (2005), URS reports (2004a, 2004b, and 2005) and Reclamation reports (2005b). These previous reports and the information developed and presented in Appendix 2B of the complete report provided the basis for estimating additional properties such as shear wave velocities, and shear and bulk modulus as shown above. The Electric Power Resource Institute (EPRI) Manual on Estimating Soil Properties for Foundation Design (1990) along with the FHWA Manual for Soil and Rock Properties (2002) were used for developing correlations of known material properties to those that were estimated.

5.3 Damping Ratio

Damping in soils is primarily hysteretic, since energy dissipation occurs when grains slide over one another. In the Mohr-Coulomb constitutive model utilized herein, due to severe dynamic loading, the internal damping generated by plastic flow represented the most important contribution to the dynamic loading. For smaller stress cycles remaining in the elastic range, 5 percent of Rayleigh damping was used in order to avoid under-damping in the elastic stress range.

Raleigh damping consists of two viscous elements. For one element, damping increases linearly with frequency (stiffness damping as a function of strain rate); for the other, damping decreases exponentially with increasing frequency (mass damping as a function of particle velocity). By choosing a center frequency, at which the combined gradients of the two curves balance out, it is possible to have damping that is nearly independent of frequency over a fairly wide spectrum on either side of the center frequency. The center frequency is usually chosen in the range between the natural frequency of the model and the predominant frequency of the input motion. The center frequencies for mid-Sea Dam and perimeter dike were estimated to be 4.0 Hz and 5.0 Hz, respectively.

5.4 Analysis Results

Nonlinear dynamic deformation analyses were performed on Models 1, 2, 3, and 4. Effects of the construction sequence were ignored. The performance of the crest of the dam and dike models are discussed with two basic parameters, the average crest displacement, which is the numerical average of the crest displacements along the crest width, 30 feet for the mid-Sea dam and 22 feet for the perimeter dike and the minimum crest displacement, which is the minimum displacement along the crest width. The average crest displacement appears to be a better parameter, based on the results of this study, for judging the adequacy of meeting the freeboard criteria and preventing and overtopping failure.

5.4.1 Model 1

Model 1 uses the full un-liquefied/un-softened properties of the mid-Sea dam; thus, the Type A core is represented by a purely frictional material with an angle of internal friction, ϕ, of 38 degrees.

With the full properties and normal polarity, the estimated lateral spread of the shell section was limited to about 7 feet on the upstream and downstream slopes of the embankment with an estimated average drop in the crest elevation of about 1.3 feet. The minimum estimated displacement at the crest was about 0.2 foot (2.4 inches). Figure C.5.4.1.7 shows the predicted crest displacements across the width of the dam along with the location of the minimum crest displacement

Using reversed polarity, the estimated maximum lateral spread was about 8.2 feet, with approximately the same drop in crest elevation.

5.4.2 Model 2

Model 2 used the liquefied or softened properties of the Type B sand/gravel shells, the stone column improved Type A sand/gravel core, and the soft lacustrine soils, as given in Table 2C.4.3. Model 2 was also the basis for the parametric study of the core and shell soil properties that will be discussed on Chapter 6.0.

These properties result in an estimated lateral spread of the sand shells of about 27 to 28 feet in the upstream direction, about 10 to 12 feet in the downstream direction, and resultant average crest vertical displacements of about 3.5 feet. The reversed polarity yielded approximately the same magnitude of vertical and horizontal displacements.

Although not explicitly modeled in the mid-Sea dam analysis, an SCB wall at the centerline of the dam would need to be capable of withstanding a shear strain of up to 0.2 percent. The maximum shear strain would occur at the interface between the constructed dam section and the underlying soils. A plot of the maximum shear strain increment along the centerline of the mid-Sea dam (Model 2) is presented on Figure C.5.4.2.10.

5.4.3 Model 3

Model 3 of the perimeter dike used non-liquefied/non-softened material properties. Modeling of the perimeter dike with the non-liquefied/non-softened material properties yielded estimated minimum crest deformations of approximately 0.1 foot (1.2 inches) and average crest deformations of approximately 0.5 foot (6 inches). Due to the confinement of the sand/gravel (Type A) core by the fine rockfill and rockfill shells and the relatively low height of the dike, the crest displacements are limited, Figure C.5.4.3.13 However, the low freeboard (0.5 foot) must be reviewed in light of the distinct possibility of overtopping, especially with the results obtained from Model 4.

5.4.4 Model 4

Model 4 of the perimeter dike used the liquefied/softened properties of the sand/gravel Type A core and for the underlying alluvial deposits. With these reduced properties, the estimated average crest settlement increased to about 1.1 feet with a minimum vertical crest displacement of about 0.6 foot. As previously stated the low freeboard of the dikes would result in overtopping with the assumptions used in Model 4.

Although not explicitly modeled in the perimeter dikes analysis, a vinyl sheet pile wall at the centerline of the dam would need to be capable of withstanding a shear strain of up to 0.2 percent. The maximum shear strain would occur at the interface between the constructed dike section and the underlying soils. A plot of the maximum shear strain increment along the centerline of the perimeter dike (Model 4) is presented on Figure C.5.4.3.13.

5.5 Summary of Deformations and Design Criteria for Optimization of Cross-Section Options

Estimates of the average and minimum vertical deformations of the four models are plotted on Figures C.3.1, C.3.2, C.3.3, and C.3.4, along with the results of the Newmark and Makdisi-Seed analyses. The assumption for both the Newmark and Makdisi-Seed analyses is that displacements occur along the failure (slide) surface that is inclined. Thus, displacements predicted using these methods contain both horizontal and vertical components. If the slide surface is steeply inclined at the crest predicted displacements can be assumed to be approximately vertical (Reclamation, 2001). For purposes of this analysis, it has been assumed that the average and minimum vertical deformations at the crest, developed from the FLAC analyses can be compared directly with the Newmark and Makdisi-Seed displacements calculated following the methodology described in Chapter 3.0.

In general, the displacements estimated with the FLAC models of two different embankment configuration options fall between the displacements estimated by the Newmark and Makdisi-Seed methods for the surface and deconvolved ground motions. The average crest displacements computed from FLAC are more centered within these limits than the estimated minimum crest displacements. Combining the FLAC results and the simplified Newmark and Makdisi-Seed results provides a sound basis to establish a planning level screening criterion for yield acceleration that can reliably and conservatively estimate adequate or marginal crest deformation performance based on the input motions as provided by the Reclamation. For purposes of "optimizing" all cross-sections, a minimum yield acceleration criterion of 0.17g was selected.

6.0 Discussion of Parametric Study Results

The purpose of the parametric study completed using Model 2 was to assist in determining the effect of different Mohr-Coulomb strength parameters on the performance of the stone column improved core of the Type A sand/gravel for the mid-Sea dam (sand dam with stone columns). It should be clear that the parameters that were varied only concern the material strengths, not the material stiffness. Material stiffness can have a more profound influence on the deformation behavior, however the stiffness behaviours of the dam and dike foundation and embankment materials have not been evaluated. $N_{1,60}$ blow counts corresponding to the estimated lower-bound of liquefied (residual) strength of soils are widely recognized and used (Seed and Harder 1990 and Seed et al., 2003). The results of the parametric study presented herein may be of use in establishing the required post-production $N_{1,60}$ blow count of the stone column improved Type A sand/gravel core materials.

Three distinct variations were performed on the stone column improved core material. First, the core material was assumed to behave as a purely frictional material, similar to Model 1. The angle of internal friction was varied from 26 to 36 degrees in 2-degree increments, while all other material parameters remained constant.

As can be seen in Figure C.6.79, the fictional assumption results in a very distinct displacement pattern. This displacement pattern can be explained as an effective stress change. Although these analyses are total stress analyses, as the effect of pore water pressure generation was not considered explicitly, effective stresses can still be computed. These stresses do not show the influence of increased pore water pressure due to seismic shaking, but they do show the difference in effective stress across the phreatic surface, as the materials on the upstream side of the phreatic surface are considered dry, and thus using the Mohr-Coulomb constitutive model to calculate the stresses, the resultant shear strength: 1) varies with depth and 2) varies with unit weight. Soils that are saturated below the phreatic surface have a buoyant unit weight whereas the soils above the phreatic surface have a moist unit weight. Also, since the tangent of the friction angle is multiplied by the normal stress, the shear strength increases with depth, unlike the next variation of parameters.

$$\tau = c + \sigma \tan\phi$$

For purely frictional materials c = 0

Where,

τ - shear stress

c - cohesion

σ - normal (vertical) stress

ϕ - angle of internal friction

$$\sigma = \gamma h$$

or

$$\sigma' = \gamma' h$$

Where,

γ - unit weight

γ' - buoyant unit weight

The second variation was also performed on the stone column improved Type A sand/gravel core material. In these cases, the core material was assumed to behave as a purely cohesive material. Cohesion values were varied from 800 to 1,600 pounds per square foot (psf) in 200 psf increments, with the base case being Model 2, which was analyzed at a value of 1,000 psf. It can be seen in Figure C.6.80 the results follow the basic assumptions of soil mechanics (FHWA 2002 and EPRI 1990), that as strengths decrease the displacements increase. Unlike the purely frictional materials, the shear strength from the Mohr-Coulomb relationship is constant for a purely cohesive material; thus, the displacement across the crest is relatively uniform, when compared with the purely frictional material, and results in greater (average and absolute) deformations than the purely frictional materials.

The final variation considered was performed on the Type B sand/gravel shell material. It is assumed that this material would be the most susceptible to (seismically induced) liquefaction and deformation and would be sacrificial. The liquefied (residual) strength of this material was assumed to be 500 pounds per square foot, pure cohesion. Three other strengths of Type B sand/gravel shell material were analyzed: 200, 300, and 400 psf, while all other values were held constant as in Model 2.

Varying the Type B shell strength yielded surprising results that appear to be counter-intuitive (see Figure C.6.81); however, the results can be explained. The results of this evaluation suggest that the lower the liquefied strength of the shell materials, the greater the lateral spread and the lower the average crest settlements would be. Although the Type B sand/gravel shells do provide a buttressing effect, it appears that the buttressing effect does not dominate the response of the crest displacements. As with most numerical methods, the program will only perform the explicitly defined calculation. Although adhesion is not explicitly included in the constitutive formulation, the fact that the finite difference grid is connected (it is a continuum), no explicit interface between different material zones exists, and the effect of the ratio of material strengths does invoke a type of adhesion. The greater the ratio of the strength of the Type A core material to Type B shell material (1,000 psf/500 psf) = 2 vs. (1,000 psf/200 psf) = 5, the less the shell material “adheres” to the core material and the less the core material “pulls” the core down with it.

7.0 Conclusions and Recommendations

Seismic deformation analyses of the optimized mid-Sea dam option (sand dam with stone columns) and its improvement of the central zone of the dam and of Reclamation's initial perimeter dikes option have been completed using the commercial finite difference code FLAC. Model cases evaluated both liquefied and non-liquefied strengths of foundation and non-densified embankment materials. The effect of a range of different material properties that would occur for various stone column improvement objectives (i.e. various target $N_{1,60}$ blow counts following densification) were also evaluated.

Study conclusions are as follows:

1. In general, the seismic-induced displacements estimated with the FLAC models of two different embankment configuration options fall between the displacements estimated by the Newmark and Makdisi-Seed methods for the surface and deconvolved ground motions. The average crest displacements computed from FLAC are more centered within these limits than the estimated minimum crest displacements. Combining the FLAC results and the simplified Newmark and Makdisi-Seed results provides a sound basis to establish a planning level screening criterion for yield acceleration that can reliably and conservatively estimate adequate or marginal crest deformation performance based on the input ground motions as provided by the Reclamation. For purposes of "optimizing" all cross-sections, a minimum yield acceleration criterion of 0.17g was selected.
2. The estimated crest deformations of the optimized mid-Sea dam (sand dam with stone columns) would generally be less than the five feet of available freeboard included in the design. To achieve this performance, the central portion of the dam would need to be densified to a minimum equivalent $N_{1,60}$ of 20, achieving a target undrained strength (S_{us}) of at least 1,000 psf, or a drained strength friction angle of at least 32 degrees.
3. The loss of freeboard predicted for the perimeter dikes is minimal. However, the FLAC analysis results suggest that a minimum of 2 feet of freeboard should be included in areas where foundation liquefaction would not occur, and 3 feet of freeboard should be included where liquefaction of the foundation materials would be expected.
5. The estimated maximum strains along the centerline axis of the dam (the location of the SCB slurry wall) would occur at the contact between the dam and the upper stiff lacustrine materials. The maximum strain occurring at this location would likely range from 0.15 to 0.2 percent. A soil-cement-bentonite (SCB) wall should be capable of withstanding this level of strain

without significant rupture and offset that could threaten the safety of the dam. Future FLAC modeling efforts should include an explicit SCB slurry wall to confirm the strain estimates of this study.

6. Once more definitive soils data, to include geophysical data, are obtained the results of this study should be reviewed, and recommendations for further study should be developed. Further studies should also incorporate the latest optimized geometry.

8.0 Limitations

This report presents the results of analyses and the conclusions in support of a planning-level study for embankment alternatives that are currently being considered for the Salton Sea restoration project. In developing the conclusions presented in this report, information used was gathered previously by others, as discussed in Chapter 2.0 of the Seepage and Slope Stability report (Appendix 2B of Kleinfelder's complete report), and interaction with Reclamation occurred regularly on the general approach to the study. Kleinfelder's experience and engineering judgment were applied during the development of the conclusions. As additional field and laboratory test data become available, the analyses and conclusions presented in this interim report will need to be reevaluated.

These analyses were conducted and this report was prepared in general accordance with geotechnical engineering practice as it exists in the site vicinity at the time of the study. No warranty, expressed or implied, is made.

This report may be used only by the client for the purposes of a planning-level evaluation of the project alternatives. Kleinfelder will not be held liable for any misuse of the information contained in this report.

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